CHAPTER 8: HYDRAULICS OF OUTLET STRUCTURES

The purpose of the outlet structure of a detention pond is to control the rate of outflow from the pond. The success or failure of a detention pond is dependent on the outlet structure. There is a wide variety of types of structures that can be used to regulate the outflow, including culverts, drop structures, and spillways. If an outlet structure is properly designed, the detention pond will provide the needed attenuation of flood flows for a range of flood frequencies, while also achieving water-quality benefits.

Outlet structures can be divided into two "hydraulic" groups: orifices/culverts and weirs. This section of the guidebook will cover the basic equations that are used in designing the outlet structure.

TAILWATER ELEVATION

Before an outlet structure can be designed, it is essential to determine the water surface elevation downstream of the structure. Ignoring the tailwater elevation is a common error that is made in determining the flow-carrying capacity of a structure. If the tailwater is not considered, the flow carrying capacity of the structure may be over -estimated (the structure would be under-sized). Under-sized structures will be subject to frequent over -topping, and may require costly revisions to the structure to prevent its failure.

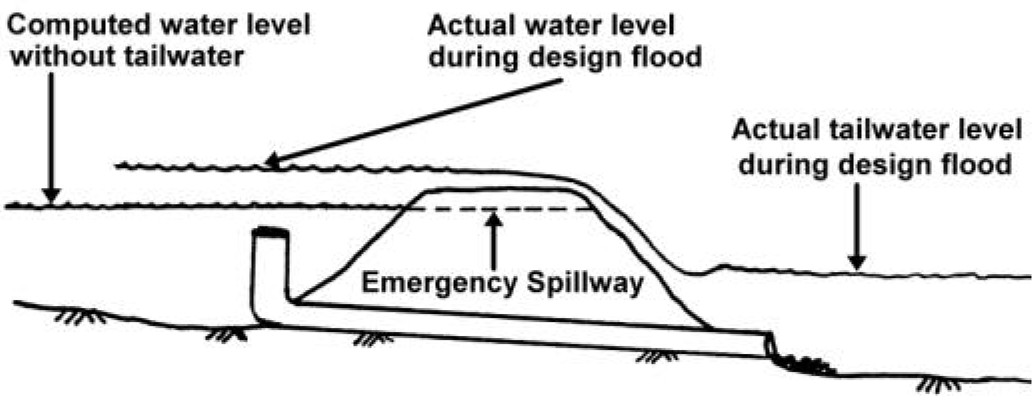


Figure 8.1 - Impacts of Not Considering Tailwater Elevation

There are various methods that can be used to determine the tailwater elevation (TWEL), including:

1. **Existing Information**

In many areas throughout Michigan, detailed hydraulic analyses currently exist. These analyses may contain water-surface elevation information for the location of interest. The primary sources of information include:

a) Flood-Insurance Studies. Prepared by the Federal Emergency Management Agency (FEMA). The studies will typically include profiles for the 10-, 50-, 100-, and 500-year floods. In addition, the hydraulic support data that was used to prepare a study is on file with the DEQ, Land and Water Management Division and with

FEMA. There are currently Flood-Insurance Studies available for about 370 communities in Michigan.

b) Flood Hazard Analyses. Prepared by the Natural Resources Conservation Service. These studies also include 10-, 50-, 100-, and 500-year flood profiles. In most instances, the hydraulic support data is on file with the DEQ Land and Water Management Division.

c) Corps of Engineers Floodplain Information. The reports will usually include 50- and 100-year profiles and in some instances a past flood event profile. The hydraulic data may not be available.

d) DEQ, Land & Water Management Division. Over the years, the DEQ has compiled a significant amount of information on various watercourses throughout the State. In most instances, the information would be limited to 100 -year elevations and possibly some stream -valley cross-section information.

e) Other sources. County or local government agencies may have information relating to hydraulic capacities of various watercourses. Some agencies that may have information could include the county drain commissioner, public works department, or the community engineering department.

2. Normal-Depth Solution

In many areas it is possible to approximate the tailwater elevation by using a "normal-depth solution." The normal-depth solution is a method of determining the water surface elevation for a given discharge at a particular location using Manning's equation referenced earlier:

$$Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2} \qquad (15)$$

where: **Q** - discharge, cfs
n - Manning's roughness coefficient
A - area of the cross section, at the given water surface elevation
R - hydraulic radius (area/wetted perimeter)
S_f - slope of the energy gradient

a) Discharge - in the earlier section, the different met hods of computing discharges were discussed.

b) n - The selection of an appropriate "n" value requires engineering judgement and experience. There are several excellent references available which provide some guideline for selection of an "n." Probably the most widely referenced book is Chow's Open Channel Hydraulics (Reference 8). Chow indicates that the roughness coefficient is a function of several factors: material, degree of irregularity, variation of channel cross-section, obstructions, vegetation, and degree of meandering. Appendix D provides a method for estimating the "n" value. Following are some typical roughness coefficients:

channel condition	n
Straight, clean channel	0.03
w/ some weeds & stones	0.04
winding channel, some weeds or stones	0.05
same as above, with some obstructions	0.06
w/ significant obstructions	0.08
overbank areas with moderate brush	0.10
w/ dense brush & grass	0.15

It is recommended that the method contained in Appendix D be used to estimate the "n" values for stream channels.

c) Area & Hydraulic Radius - The area and hydraulic radius can be obtained from the stream-valley cross section. The **area**, or waterway area is the portion of the cross section that will be carrying flow. The **hydraulic radius** is defined as the area divided by the wetted perimeter.

   The **wetted perimeter** is the wetted surface of the cross section which causes resistance to flow. When the cross section is broken up into sub-sections to define different roughness coefficients, the water-to-water interface is not included when computing the wetted perimeter. (See figure 8.2).

d) Energy Gradient (S_f) - The **energy gradient** (friction slope) can be thought of as the slope of the water surface. For most normal depth solutions, the slope must be determined. The slope may be estimated from:

   1) U.S.G.S. quadrangle. . The contour angle on a quadrangle limit the accuracy of determining the friction slope. As a result, this method should be used when no other method is available. However, topographic maps that have 1-, 2-, or 4-foot contour intervals can provide a reasonable estimate of the friction slope.
   2) Slope of the Water Surface at the Time of Survey. When survey information is being obtained at the site, it should include water surface elevations at several downstream locations. Judgement is needed to determine how far downstream to take elevations. As a rough guide, the elevations should be taken about every 100 feet for about 300 feet downstream. Once the distance and change in water surface elevation is known, an estimated slope may be used in the normal depth solution. It is very likely that the measured slope will not be uniform. Some judgement must be used to get a reasonable average of the slope.
   3) Slope of High-Water (Flood) Profile. The best method of estimating the friction slope of a watercourse is to obtain high-water marks along the stream. A considerable amount of information can be obtained from talking with long-time residents. However, actually being on-site during a flooding event can provide a significant amount of insight into how a watercourse will function under flooding conditions. During high water, the water-surface elevation slope may be different than would be occurring during low flow. During these high stages, the flow may begin to encounter obstructions that will increase the resistance to flow.

Once the slope of the water-surface elevation is estimated, Manning's formula (equation 15) can be used to calculate the water-surface elevation for a given discharge. This water-surface elevation will be the "tailwater elevation" at the outlet structure.

e) Normal-Depth-Solution Computations. In most cases, the cross section that has been taken downstream of the outlet structure will consist of a channel, a left overbank, and a right overbank (see figure 8.2). In addition, the "n" value will not likely be constant across the entire cross section. To compute a normal depth solution, it would be necessary to break the cross section up into subsections based on "n" values.

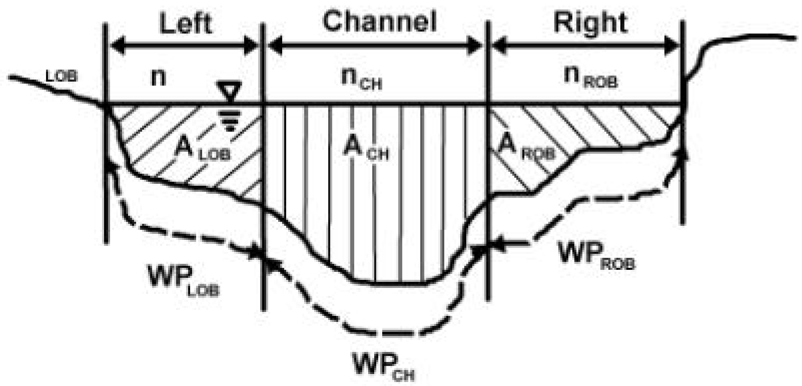


Figure 8.2 - Cross Section with More than One "n" value

The procedure for determining the water surface elevation for a given discharge is a trial-and-error process:

1. Assume a water surface elevation and compute the hydraulic properties for each sub-section (area, wetted perimeter, and hydraulic radius).

2. Compute the discharge (Q) for each subsection of the cross section at the assumed water surface elevation using Manning's formula:

   $$Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2}$$

3. Combine the discharges for each of the sub -sections to obtain the total d ischarge for the cross section at the assumed water-surface elevation.

   $$Q_t = Q_1 + Q_2 + Q_3$$

4. Plot the water surface elevation (stage) versus discharge on a graph, such as shown on figure 8.3.

5. The process is repeated until the graph defines the "rating curve" for the cross section.

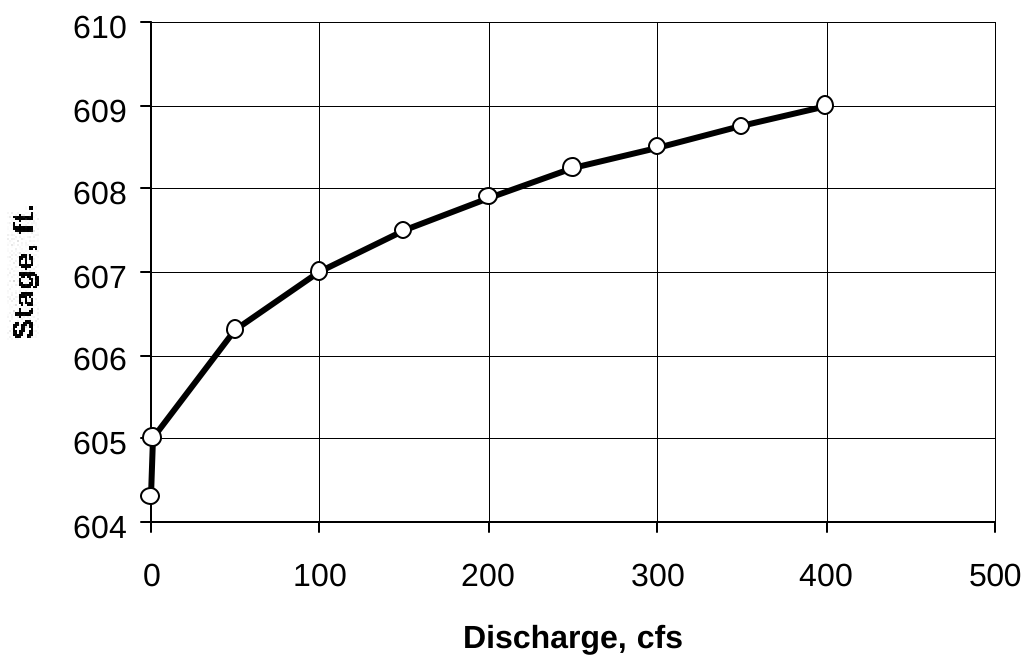


Figure 8.3 - Typical Rating Curve

Once the rating curve is developed downstream of the outlet structure, it is possible to determine the water-surface elevation for a given discharge. The tailwater-elevation information is essential in defining the outlet-structure rating curve.

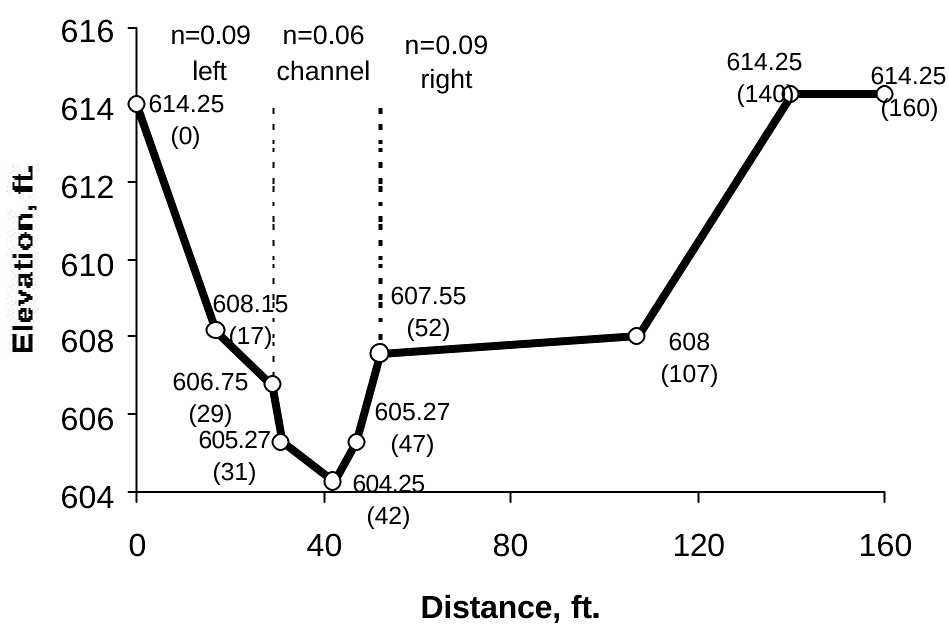


Figure 8.4 - Example Cross Section

Following is an example of a normal depth solution:

Example 8.1: Given a discharge of 345 cfs, a stream slope of .004 ft/ft, and the following cross section, compute the normal depth solution.

Solution:

– Assume a water surface elevation - 608.7

– Compute hydraulic properties for each "subsection" of the cross section:

– The area (A) and wetted perimeter (P) can be computed for the assumed water surface elevation of 608.7, by using simple algebra (see figure 8.4). The hydraulic radius (R) is defined by Area (A)/wetted perimeter (P).

  left overbank: n=.09; A=15.4 sq. ft, P=13.7 ft, R=1.13 ft

  channel: n=.06; A=79.9 sq. ft, P=24.1 ft, R=3.32 ft

  right overbank: n=.09; A=52.2 sq. ft, P=58.8 ft, R=.89

– Compute Q for each subsection, using Manning's equation (15), sum all of the Q's to obtain a total Q:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

Qleft:	17 cfs
Qchannel:	278 cfs
Qright:	50 cfs
Q total:	**345 cfs**

– If a range of tailwater elevations and discharges are required, additional elevations can be selected, and discharges computed.

elevation	604.25	606.00	606.30	607.00	607.30	608.00	609
computed Q, cfs	0	34	50	95	120	200	425

– Plot results on a stage versus discharge curve (See figure 8.5).

  The graph can be used to determine an elevation for a given Q.

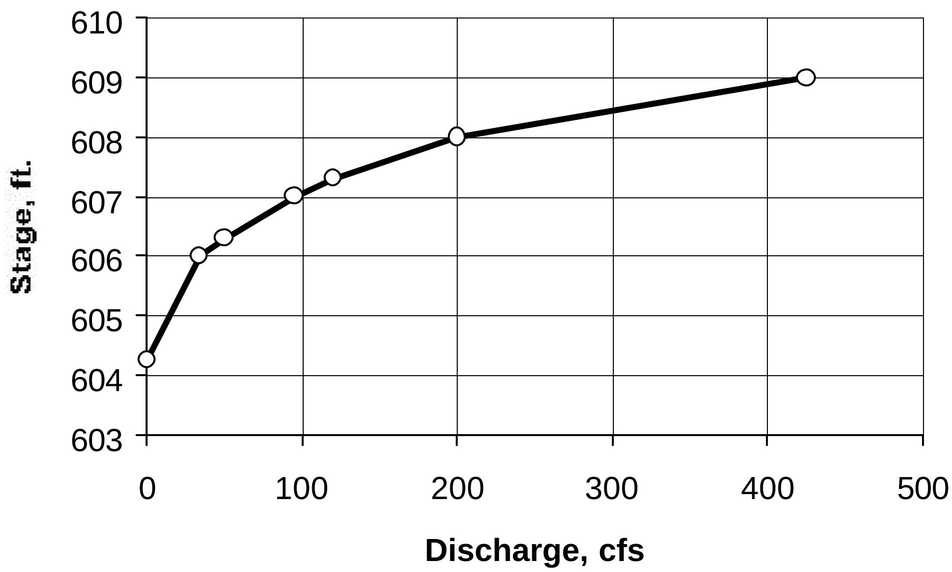


Figure 8.5 - Rating Curve for Example

Important Note:

A normal depth solution is applicable only if there are no downstream restrictions which may cause a backwater at the site (see figure 8.6). Restrictions can include bridges, culverts, dams, and natural and man -made restrictions. In addition, the stream slope should be fairly uniform, and the field-surveyed cross sections must be representative of the stream. The cross sections must be taken perpendicular to the flow as it would occur during flood conditions.

3. **Detailed Hydraulic Analysis**

As noted above, a normal-depth solution is not appropriate in all instances. For those times when a downstream restriction is causing a backwater at the site, a more in -depth analysis is required. Such an analysis can be done using computations by hand (which is quite tedious) or through the use of one of numerous computer programs (Some of the computer programs available include HEC-RAS (reference 53), HEC-2, WSPRO, and WSP-2).

4. **Equivalent Hydraulic Grade Line**

If no other information is available, it is possible to approximate the tailwater elevation by using the equivalent hydraulic grade line:

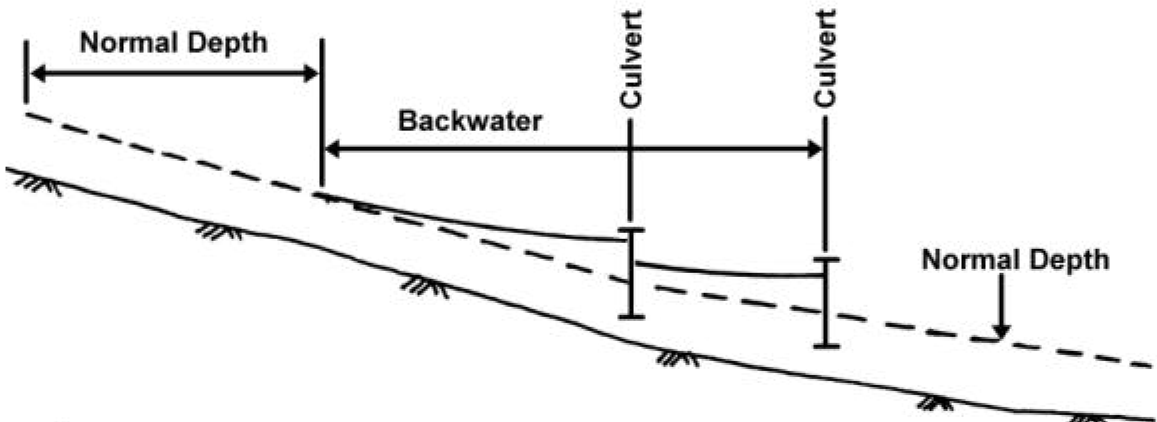
$$ho = (D + d_c)/2 \qquad (29)$$

where: **ho** - the vertical dimension from the culvert invert to the outlet equivalent hydraulic grade line.
D - diameter of the culvert
d_c - critical depth in the culvert for the given discharge

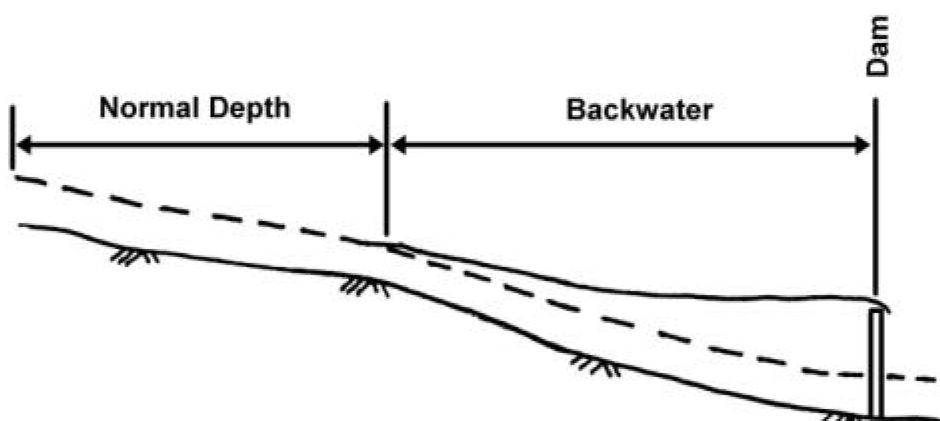
Critical depth is defined as the minimum specific energy for a given discharge. Specific energy being the total energy head ($Y + v^2/2g$) above the culvert invert or channel bottom. For depths of flow greater than d_c, the flow is sub-critical, or tranquil. If the depth of flow is less than d_c, the flow is super-critical, or rapid. For a constant discharge, as the depth decreases, the flow area decreases, which results in increased velocity. On the other hand, as the depth increases, the flow area increases, and the velocity decreases. (Figure 8.7 shows a plot of the depth versus specific energy for a constant discharge.)

Preferably, the hydraulic grade line should not be used in place of tailwater computations. It should be used as a comparison to the tailwater determined by computations. The equivalent hydraulic grade line should be used if it exceeds the tailwater elevation computed by other methods.

Figure 8.8 shows a typical chart for determining d_c for a given culvert and discharge. Additional charts are included in Appendix E.

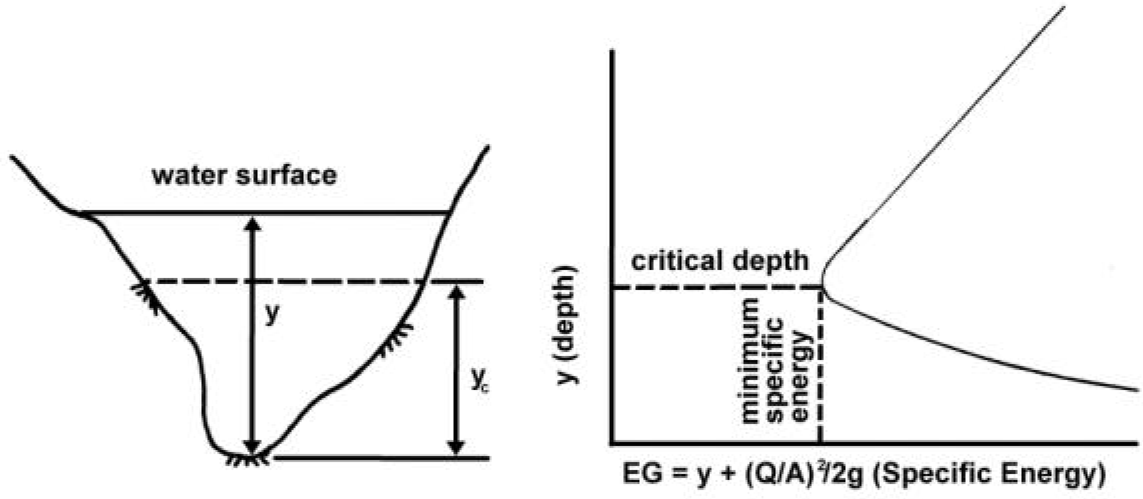


a. Backwater Effects Due to Downstream Culverts/Bridges



b. Backwater Effects Due to a Downstream Dam

Figure 8.6 - Effects of Backwater Due to Downstream Restrictions



a. Typical Cross Section **b. Specific Energy Diagram**

Figure 8.7 - Critical Depth

Example 8.2: Given: a 5-foot diameter corrugated metal pipe with an outlet invert elevation of 580.2 feet. The total discharge is 250 cfs, and the computed tailwater elevation is 584.0 feet. Compute the equivalent hydraulic grade-line.

1. From figure 8.8, for a discharge of 250 cfs d_c is 4.4 feet.
2. ho = (D + d_c)/2 = (5 + 4.4)/2 = **<u>4.7</u>**
3. The equivalent hydraulic grade-line elevation:

   Outlet invert + ho = 580.2 + 4.7 = **<u>584.9</u>**

In example 8.2, the equivalent hydraulic grade line exceeds the tailwater elevation of 584.0 feet that was computed. The "tailwater" elevation that should be used in the analysis of the culvert for the discharge of 250 cfs is 584.9 feet and not 584.0 feet.

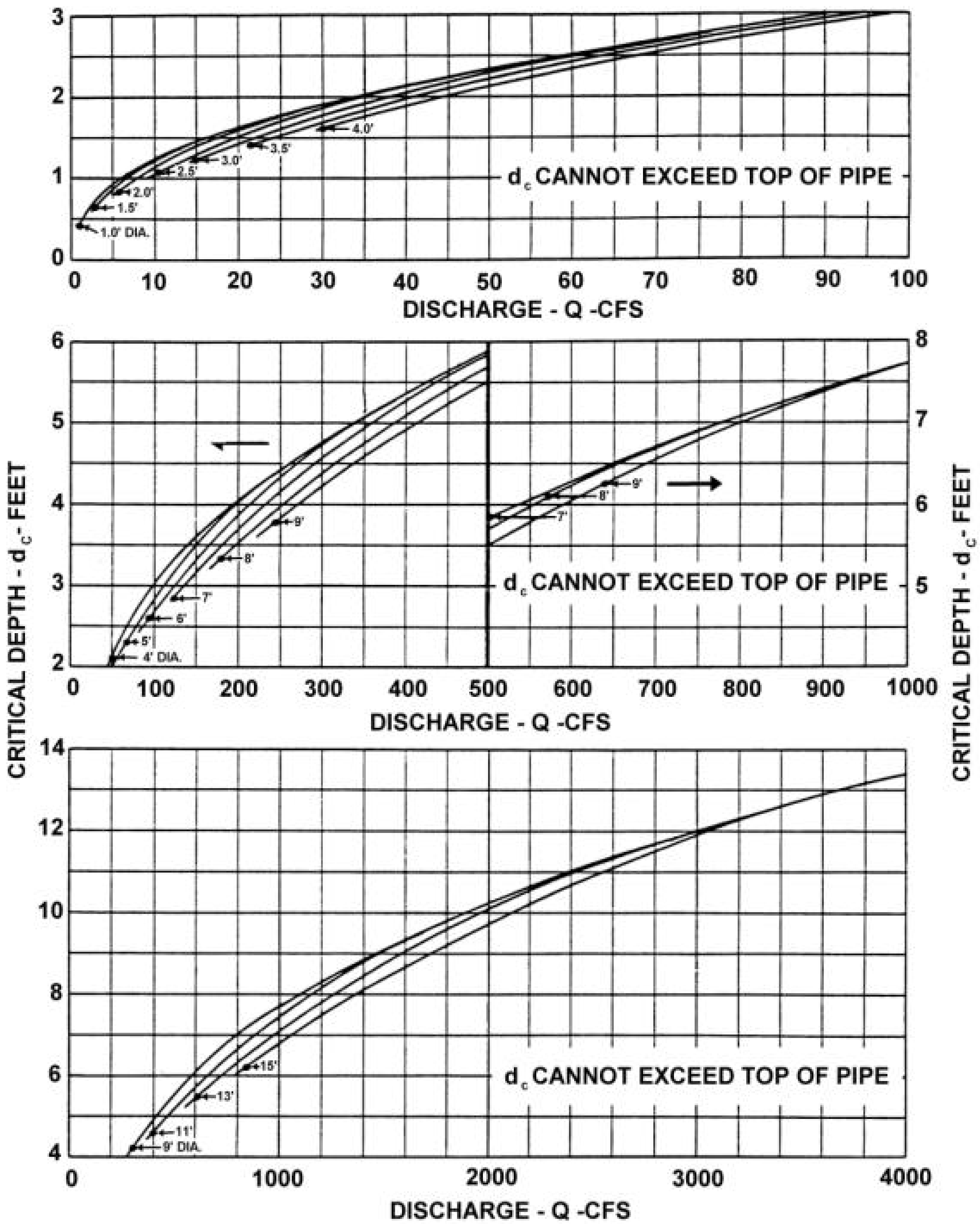


Figure 8.8 - Critical Depth Chart for Circular Pipe

(Source: FHWA, 1985, reference 13)

High-water marks

Hydraulics and hydrology involve considerable judgement on the part of the designer in estimating the various coefficients that are needed to determine the discharge and the anticipated water-surface elevation. Since these are not exact sciences, the importance of comparing the preliminary results with what has occurred in the past cannot be stressed enough. High-water marks are a good indicator of how the watercourse will function during a flood.

The validity of the computations is destroyed if the "100-year" elevation that has been computed is exceeded each spring. If high-water marks do not support the water surface elevations determined by the computations, the friction slope (S_f) and "n" values should be reviewed, as these are the values which typically induce errors into the normal-depth solution.

INLET & OUTLET CONTROL - CULVERT FLOW

Before the basic orifice equation is discussed, it is necessary to get a brief overview of culvert (orifice) flow, which is be classified into two categories:

1. Inlet Control (figure 8.9)
2. Outlet Control (figure 8.10)

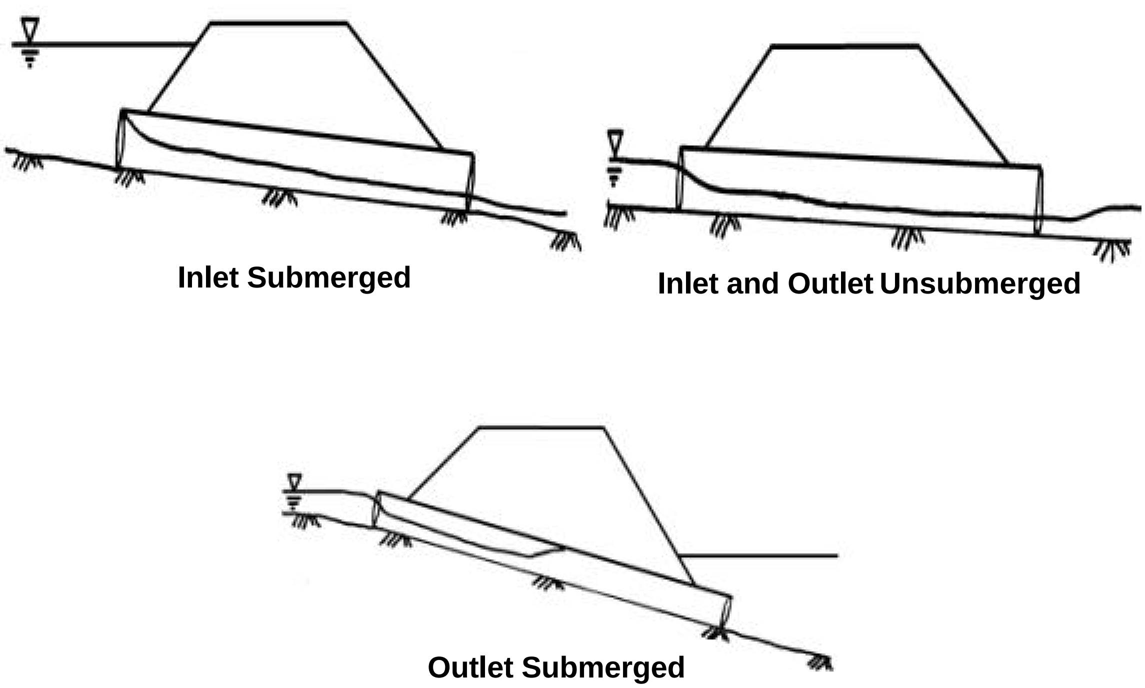


Figure 8.9 - Culverts Flowing Under Inlet Control

Inlet Control

The capacity of a culvert that is flowing under <u>inlet</u> <u>control</u> is governed only by the cross-sectional area of the culvert and the inlet configuration (sharp-edge inlet, rounded inlet, a headwall, mitered, or protruding from fill). The tailwater and length of the culvert have no impact on the upstream water surface elevation (In other words, the water can get out and away from the culvert faster than it can get into the culvert.)

Inlet control conditions will typically occur for culverts that are fairly steep, or for an inlet that is very restrictive.

Outlet Control

For culverts flowing under **outlet control**, the upstream water surface elevation is controlled by a combination of:

1. Downstream water-surface elevation
2. Culvert length
3. Culvert material
4. Culvert slope
5. Inlet configuration (sharp, rounded wingwalls)

The downstream water-surface elevation (tailwater) can be controlled by downstream restrictions or the flow-carrying capacity of the channel. The tailwater is a very critical factor in the design of an outlet structure. Earlier, a method of computing a tailwater elevation was discussed.

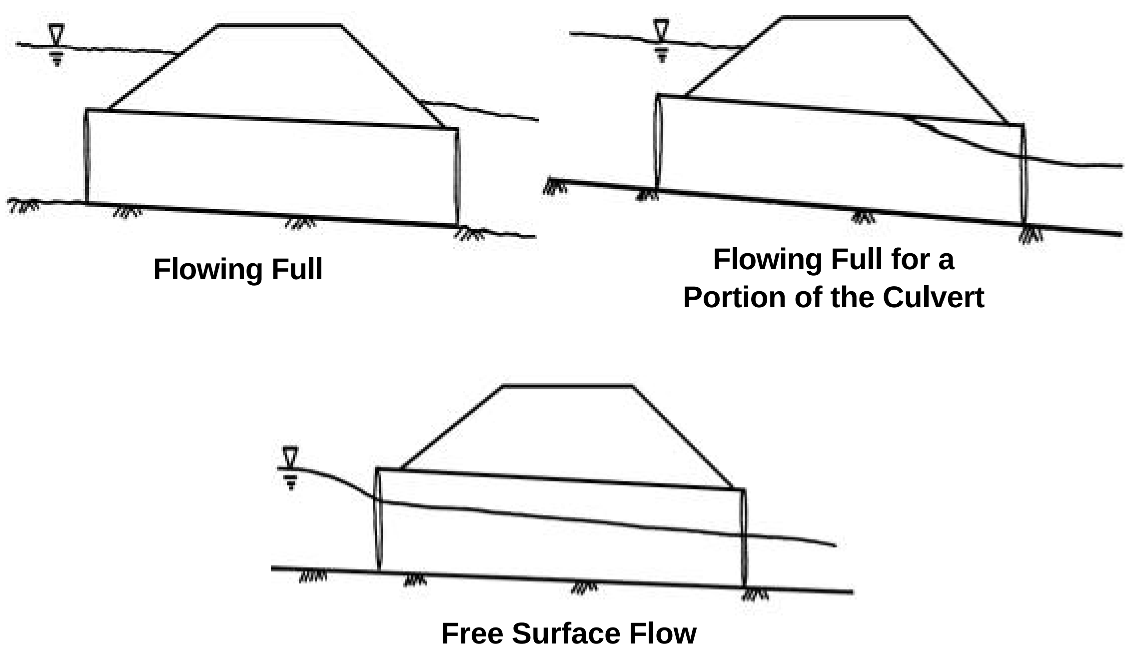


Figure 8.10 - Culverts Flowing Under Outlet Control

In some instances, it is not readily apparent whether a culvert is functioning under inlet or outlet control. For this reason, it is highly recommended that the outlet structure be analyzed for both inlet and outlet control to ensure that the structure will be functioning as designed.

Orifice/Culvert Flow

An orifice can be in the form of a pipe or box culvert or some other opening. The basic equation for orifice flow is given by:

$Q = CA\,(2gH)^{1/2}$ (reference 7) (30)

where:
- **Q** - discharge (outflow) in cubic feet/second
- **C** - discharge coefficient
- **A** - cross sectional area of orifice, square feet
- **g** - acceleration due to gravity, 32.2 ft/sec/sec
- **H** - head on the orifice, feet

Note: See figures 8.11a and 8.11b for the definition of head. For free flowing outlet (figure 8.11a), **H** is the difference in elevation between the upstream water surface elevation, and the center of the orifice.

For submerged flow, **H** is the difference between the upstream and downstream water surface elevations (figure 8.11b).

Equation 30 is arranged so the discharge can be computed for a given H. It is possible to rearrange (30) to a form in which it is straightforward to compute H for a given Q.

$H = (Q/CA)^2/2g$ (31)

In equations 30 & 31, the C coefficient is obtained from reference material. The discharge coefficient (C) is a function of many variables such as size, shape and sharpness of the opening, the type of material the orifice is made of, head on the orifice, and the extent to which the flow is already suppressed.

a. Free Flowing Outlet **b. Tailwater-Influenced Outlet**

Figure 8.11 - Head Differential

The C coefficient can range from .15 for a long corrugated metal pipe, to 0.95 for a short tube (orifice) with rounded edges. A typical value for C would be about 0.6, for sharp edged orifices. Values of C can be found in Brater and King "Handbook of Hydraulics" (reference 7). If no references are available, the following C coefficients may provide some guidance:

Table 8.1 - C coefficients for orifices and culverts

orifices		C
	Sharp opening	0.60
	suppressed on sides and bottom	0.70
	suppressed on sides, top, and bottom	0.90
	Orifice with rounded corners	0.95
culverts		
	4-ft CMP no headwall, 60 ft long	0.54
	4-ft CMP with headwall, 60 ft long	0.57
	4-ft concrete pipe, 60 ft long, square edge	0.75
	4-ft concrete pipe, 60 ft long, beveled edge	0.82

The following equation is another form of equation 31:

$$H = K\,(Q/A)^2/2g \qquad (32)$$

where: **K** is a total loss coefficient which can be computed by knowing the length of the culvert, the culvert material, and the entrance geometry (references 12 & 45).

$$K = 1 + k_e + k_f \qquad (33)$$

where: $\mathbf{k_e}$ - entrance loss coefficient (Table 8.2)
$\mathbf{k_f}$ - friction loss coefficient:

$$k_f = \frac{29.1 n^2 L}{R^{4/3}} \qquad (34)$$

where: **n** – Manning's roughness coefficient (table 8.2)
L - length of culvert
R - hydraulic radius, (area/wetted perimeter)

Outlet Control, Full or Partly Full Entrance Loss Coefficients

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
* End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch. Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
* End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel square-edged at crown	0.5
Wingwalls parallel (extension of sides) square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

Table 8.2 - Entrance Loss Coefficients

(Source: reference 12)

Table 8.3 - Selected Manning's n Values

~~Material~~			~~Manning n~~
Concrete Pipe:	w/ smooth walls		.011 - .013
	w/ rough walls		.015 - .017
Concrete Box:	w/ smooth walls		.012 - .015
	w/ rough walls		.014 - .018
Corrugated Metal Pipes and Boxes:			
	Annular corrugations	2-2/3" x 1/2"	.027 - .023
		3" x 1"	.028 - .027
		6" x 2"	.035 - .032
	Helical corrugations	2-2/3" x 1/2"	.012 - .024

(Source: FHWA, reference 12)

Substituting equation 34 into equation 33:

$$K = 1 + k_e + \frac{29.1n^2L}{R^{4/3}} \qquad (35)$$

To demonstrate how this equation above may be used, the following example is given.

Example 8.3: Compute the K value for a 3 -foot circular concrete pipe, 50 feet long, with a square-edge entrance. Using the computed K value, determine the head needed to carry 70 cfs. (The tailwater submerges culvert outlet.)

For a 3-foot circular concrete pipe:

Area (A) $= Pi(D)^2/4 = 3.14(3)^2/4 = 7.07$ square feet

Wetted Perimeter (WP) $= Pi(D) = 3.14(3) = 9.42$ feet

Hydraulic Radius (R) $= A / P = 7.07ft^2/9.42\ ft = 0.75$ feet

Use equation 35 to compute total loss coefficient:

$$K = 1 + k_e + \frac{29.1n^2L}{R^{4/3}}$$

1. From table 8.2, **ke = 0.5;** From table 8.3, n = 0.012

2. $kf = \frac{29.1\,(n)^2L}{R^{4/3}} = \frac{29.1\,(0.012)^2\,50}{0.75^{4/3}} = \mathbf{.31}$

3. $K = 1 + ke + kf = 1 + 0.5 + 0.31 = \mathbf{1.81}$

4. To compute the head needed to pass the given discharge (Q), equation 32 is used:

$H = K(Q/A)^2 / 2g$

Q = 70 cfs	(given)
K = 1.81	(computed in step 3)
A = 7.07 sq. ft.	(area of 3-foot pipe)
g = 32.2 ft./sec./sec.	(gravity constant)

$H = 1.81(70/7.07)^2/64.4 = 1.81(9.9)^2/64.4 = \underline{\mathbf{2.76}}$ ft.

The computed H is the head differential between the tailwater and the headwater.

If the tailwater elevation is 584.0 feet, the outlet control computations produce an upstream stage equal to:

H + TWEL = (2.76 + 584.0) = **586.76 ft.**

In addition to obtaining C values from tables, or computing K values, it is possible to use nomographs that have been developed for most types of culverts and pipes. Figure 8.13 is an example of a nomograph. The nomograph allows the designer to determine a head (H) for a given discharge or vice-versa. Using example 8.3 above, the outlet control nomograph (figure 8.13) indicates H as 2.76. The "H" that was computed using the equation or the nomograph is added to the tailwater elevation to determine the upstream water surface elevation (see figure 8.12).

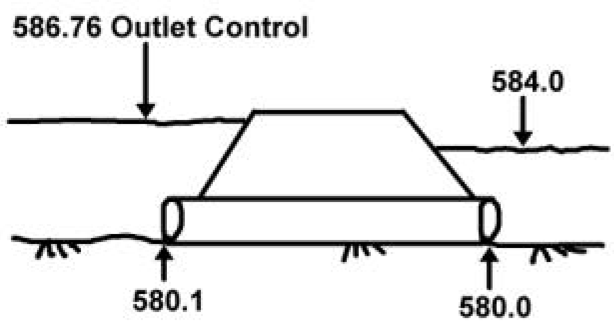


Figure 8.12 - Head Differential for Culvert Example

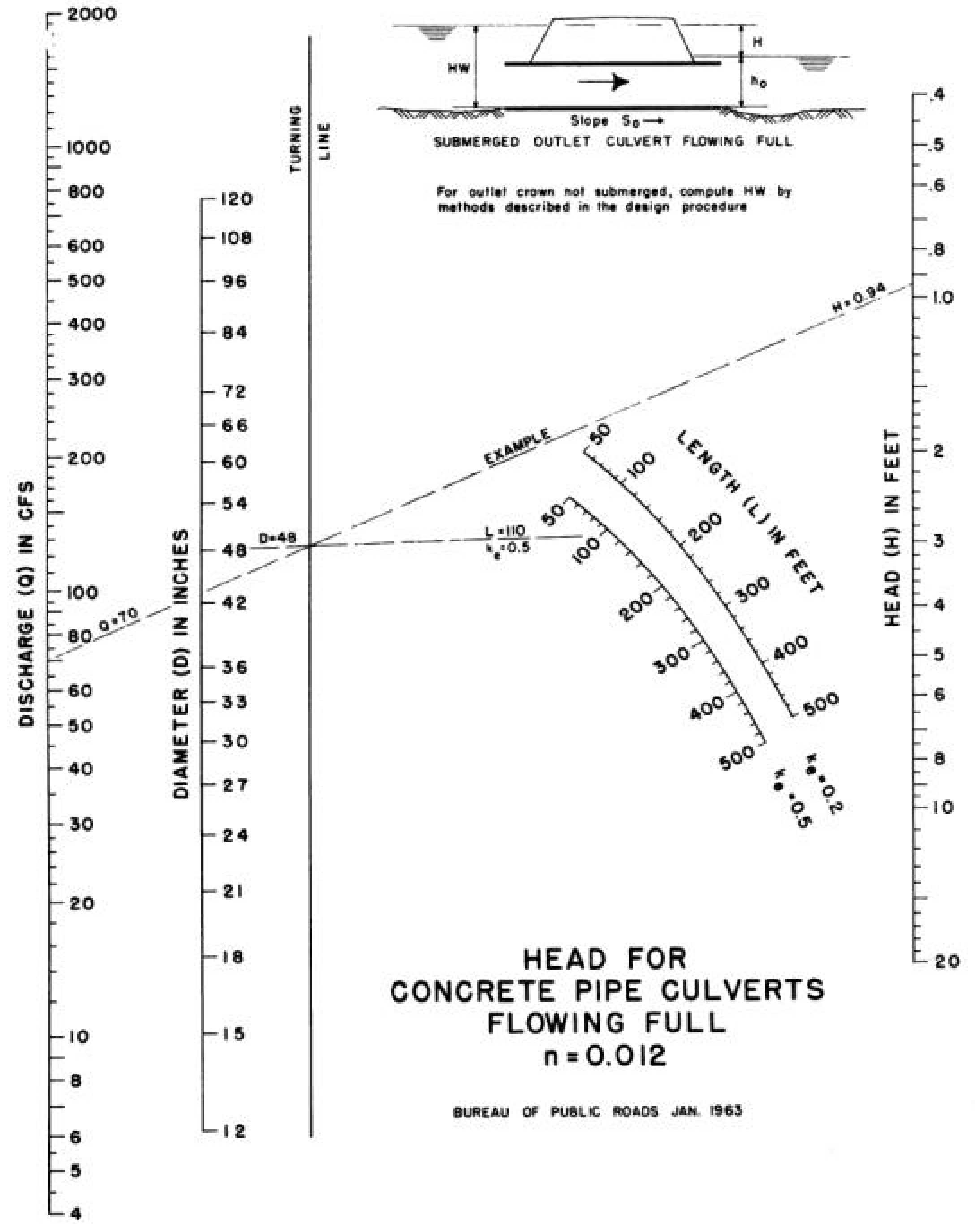


Figure 8.13 - Outlet-Control Nomograph

(Source: reference 12)

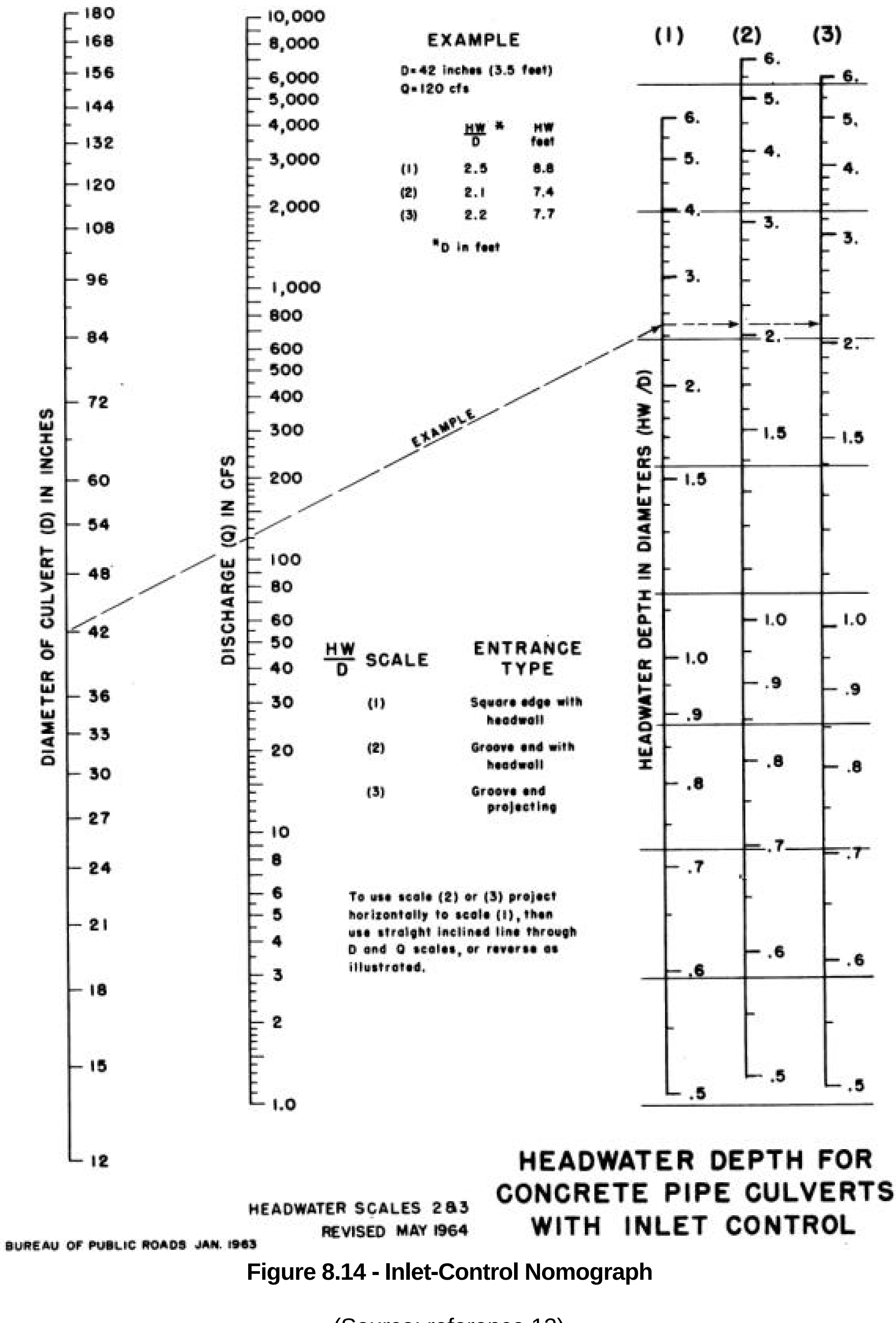


Figure 8.14 - Inlet-Control Nomograph

(Source: reference 12)

It was noted earlier that a culvert can function under either inlet or outlet control. The head (H) of 2.76 feet computed in the example 8.3 was determined assuming outlet control. It is also necessary to determine the upstream stage due to inlet control. Figure 8.14 is a typical inlet control nomograph for a culvert. Other nomographs are available in reference 12.

Example 8.4: Inlet-control computations

Given a 3-foot circular concrete pipe with a square edge entrance, determine the head needed to carry 70 cfs, for inlet control.

Using the nomograph in figure 8.14:

HW/D = 2.1 (Q = 70 cfs, Diameter = 36 inches)

where: **HW** - headwater in feet above the upstream invert of the culvert
D - diameter of the culvert

thus : **HW** = 2.1(D) = 2.1(3.0) = **6.3 feet**

If the upstream invert elevation of the culvert is 580.1, inlet control would produce an upstream stage was equal to:

HW + Upstream invert elevation = 6.3 + 580.1 = 586.4 feet

See figure 8.15 for a comparison of upstream stages for inlet and outlet control for examples 8.3 and 8.4.

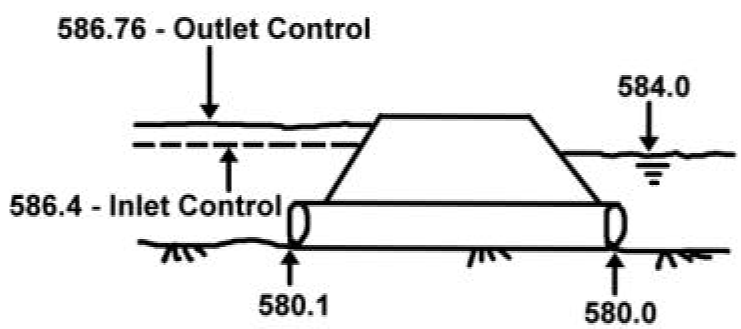


Figure 8.15 - Comparison of Inlet and Outlet Control for Example

In this example, the stage produced under outlet control exceeds the stage produced under inlet control, Thus, the culvert is functioning under outlet control. If tailwater conditions were different, or if a different discharge was analyzed, the control could shift to inlet control.

A form has been developed by the Federal Highway Administration to be used to analyze a culvert. The form (figure 8.16) is organized to allow the designer to determine both inlet and outlet control stages.

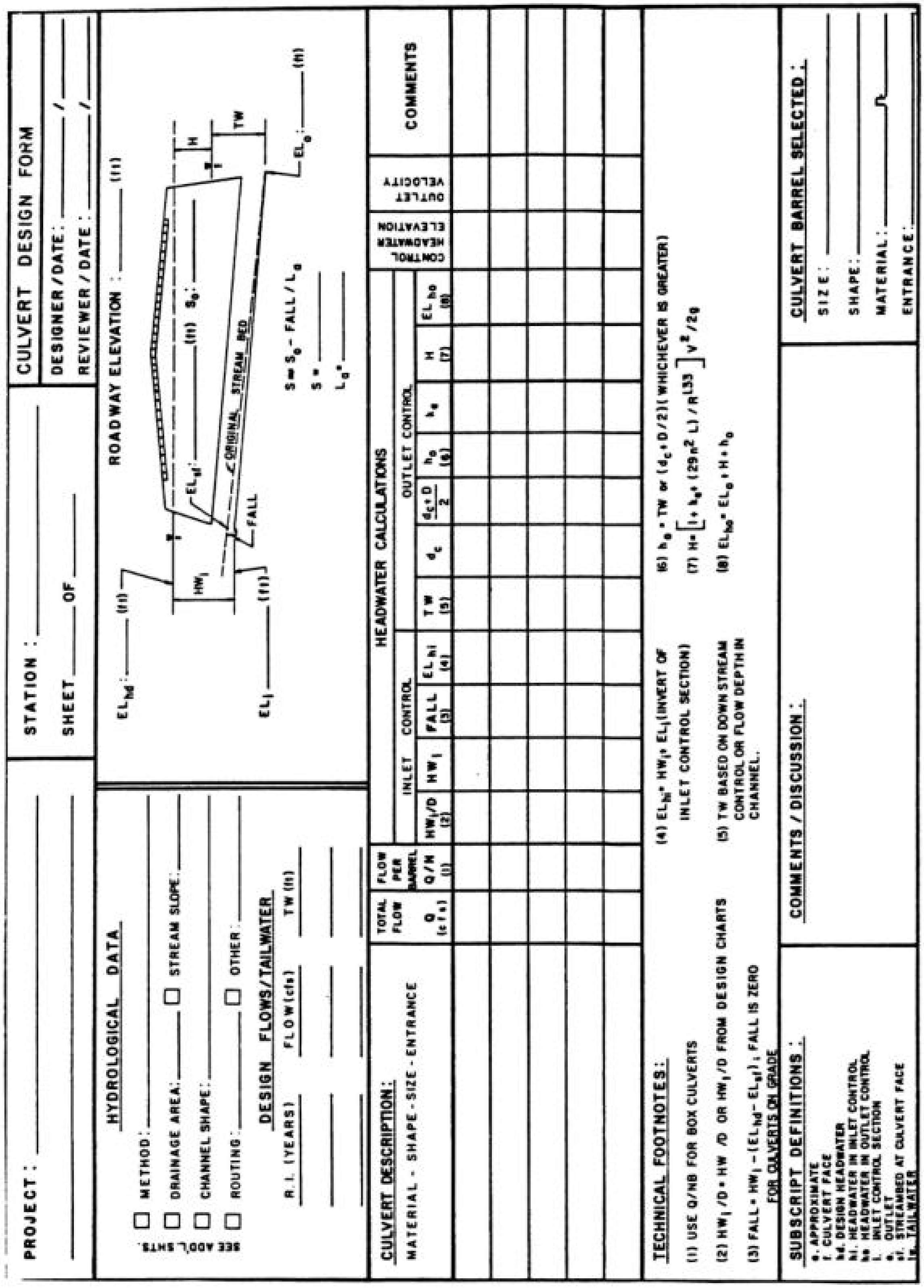


Figure 8.16 - Culvert Design Form
(Source: reference 12)

Low Flow (Free surface flow)

If the culvert is flowing greater than 75% full, the procedure outlined earlier provides reasonable results. In some instances, the capacity of the culvert will be much greater than the discharge, in which case, the culvert will function as an "open channel" (see figure 8.17).

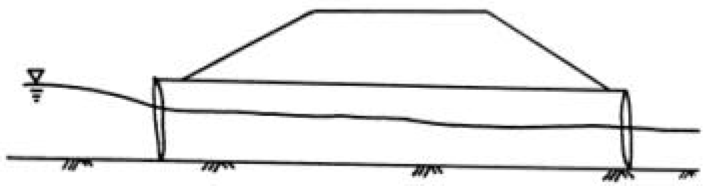


Figure 8.17 - Culvert Functioning Under "Free-Surface" Flow

To estimate the losses through a culvert functioning as an open channel, it is possible to use equations 32 and 35. Where:

$$K = 1 + k_e + \frac{29.1\, n^2 L}{R^{4/3}} \tag{32}$$

$$H = K\,(Q/A)^2\,/2g \tag{35}$$

For culverts flowing partially full, the hydraulic radius (R) and area (A) are adjusted to account for the culvert not flowing full.

As an example, if the culvert in figure 8.17 were a 4-foot by 4-foot concrete box culvert 50 feet long with a square edge headwall and a tailwater depth of 2.5 feet, the adjusted hydraulic radius, area, and K value would be computed as follows:

Area(adjusted) = width x tailwater depth = 4 ft. x 2.5 ft. = 10 ft ^{2}

Hydraulic Radius (adjusted) = wetted perimeter$_{adj}$/area$_{adj}$

= (2 x depth + width)/ 10 ft ^{2}

= (2 x 2.5 ft + 4 ft)/ 10 ft ^{2}

= 1.125 ft.

Adjusted K $= 1 + 0.5 + \frac{29.1(0.012)^2(50)}{(1.125)^{4/3}} = 1.67$

To compute the head loss through the culvert, equation (35) would be used after inserting adjusted K and A values.

$$H = K_{adj}\,(Q/A_{adj})^2/2g = 1.67\,(Q/10)^2/64.4$$

Appendix F contains information needed to compute adjusted areas and wetted perimeters for circular and pipe-arch culverts.

To accurately determine the losses through a culvert that is going "low flow," step-backwater computations would be required through the culvert. The computations would include the energy losses due to friction and the entrance losses. There are computer and hand-held-calculator programs available for step -backwater computations. One such computer program is a culvert analysis program, **HY-8**, which was developed by Pennsylvania State University in cooperation with the Federal Highway Administration (FHWA). This program can be downloaded from the DEQ homepage at http://www.michigan.gov/deq/0,1607,7-135-3313_3684_15299-11304--,00.html. Even if a computer program is used to design an outlet structure, it is essential for the designer to be able to review the results to determine if they are reasonable.

Additional nomographs and more information on culvert flow is contained in "Hydraulic Design of Highway Culverts" (reference 12). The publication may be obtained through:

National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Road
Springfield, VA 22162
(703) 487-4600

Weirs

The other type of outlet structure is classified as a weir. Weirs can be used as the primary outlet structure, or they can function as the emergency spillway. Weirs can be broken into two categories: sharp crested and broad crested.

Sharp-Crested Weir

A sharp-crested weir (see figure 8.18) has a "sharp" upstream corner. The flow over a sharp-crested rectangular weir is defined as:

$$Q = CLH^{3/2} \qquad (36)$$

where:
- **Q** - discharge, cfs
- **C** - discharge coefficient
- **L** - effective length of the weir, ft.
- **H** - head, difference in feet between the crest of the weir and the upstream water energy gradient.

The discharge coefficient (C) for the sharp-crested weir can be calculated using:

$$C = 3.27 + \frac{0.4h}{P} \qquad (37)$$

where :
- **P** - height of weir crest above the channel bottom
- **h** - head of energy gradient above crest

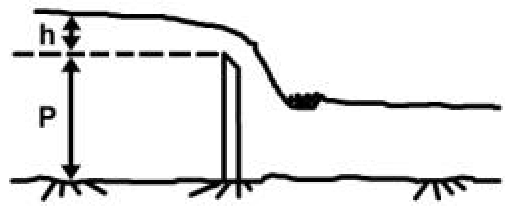


Figure 8.18 - Sharp-Crested Weir

The effective weir length (L) may have to be corrected to account for end contractions (see figure 8.19). End contractions refer to a weir configuration in which flow must contract to get over the weir.

$$L = L' - 0.1nh \qquad (38)$$

where: **L'** - measured weir length
n - number of end contractions
h - head of energy gradient above crest

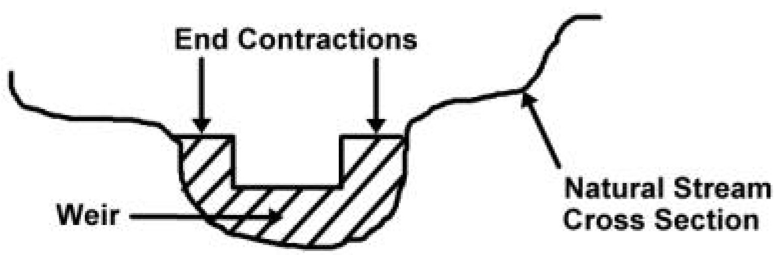


Figure 8.19 - End Contractions For Sharp-Crested Weir

Following is an example of discharge computations for a sharp crested weir.

Example 8.5: Given a sharp-crested weir, with a crest that is 4 feet above the channel bottom. The measured weir length is 15 feet with contraction on each end of the weir. Compute the discharge for a head of 2 feet.

1. Compute C: Equation (37) $C = 3.27 - 0.4h/P$
   $= 3.27 - 0.4(2/4)$
   $= 3.07$

2. Compute L: Equation (38) $L = L' - 0.1nh$
   $= 15 - 0.1(2)2$
   $= 14.6$ ft

3. Compute discharge: Equation (36) $Q = CLH^{3/2}$

   $Q = 3.07(14.6)(2)^{3/2}$ = **127 cfs**

Triangular sharp-crested weir

The triangular weir is able to provide very accurate measurement of flows. Flow over the triangular weir is defined as:

$$Q = 2.5 \tan (a/2)\, H^{2.5} \qquad (39)$$

where: **a** - angle of the notch of the weir (figure 8.20)

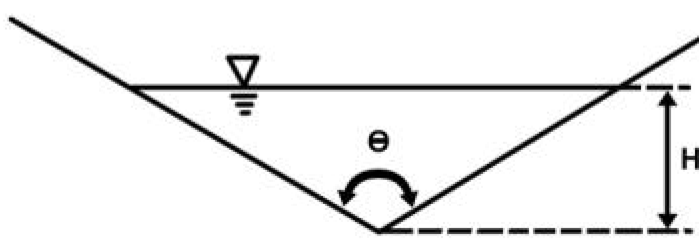


Figure 8.20 - Triangular Weir

Example 8.6: Given a= 120 degrees, and H = 3.0 feet.

Compute the discharge.

Equation (39) $Q = 2.5 \tan (120/2)\, (3)^{2.5}$

$= \underline{67.5}$ $\underline{\text{cfs}}$

Broad-Crested Weir

A broad-crested weir differs from a sharp-crested weir in that the weir is wide enough to support the water as it flows over the weir (see figure 8.21). The discharge over a broad - crested weir is determined using:

$$Q = CLH^{3/2}$$

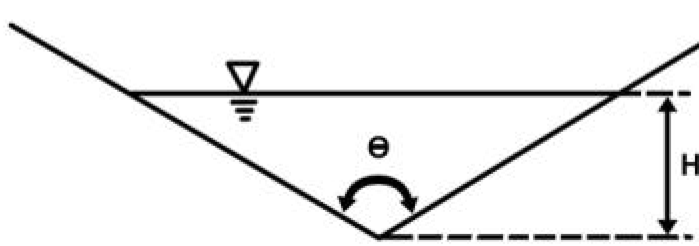


Figure 8.21 - Broad-Crested Weir

Discharge Coefficients (C)

The discharge coefficient can be obtained from figure 8.24. Depending on the shape of the weir and amount of head on the weir, the C value may range from 2.4 to 3.1. If the head greatly exceeds the width of the weir, the broad-crested weir will function as a sharp-crested weir, and the C may reach 3.32. For highway/dam embankments a C value of 2.8 to 3.0 is generally used.

Weir Length (L)

The length of the weir that is used in the equation should be reduced for any obstructions or high ground that may reduce the efficiency of the weir (see figure 8.22). Typically, broad-crested weirs will be relatively long. As a result, end contractions should not be a major factor. Equation (38) can be used to reduce the effective weir length due to end contractions.

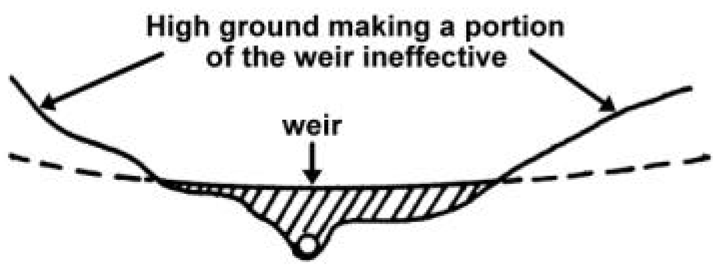


Figure 8.22 - Weir length Ineffective Due to High Ground

Submergence of Weir

In addition to impacting the flow-carrying capacity of culverts, high tailwater can also impact weirs. The weir equations that have been shown assume that the weir is free flowing. However, if the tailwater rises high enough, the weir will be submerged, and the weir flow-carrying capacity will be reduced (see figure 8.23). If submergence occurs, the weir calculations will have to be adjusted to account for the high tailwater. For <u>sharp-crested</u> weirs, the discharge can be adjusted for submergence using the following equation:

$$Qs/Q = [1.0 - (hs/h)^n]^{0.385} \qquad (39)$$

where:
- **Qs** - discharge for a submerged weir
- **Q** - discharge computed using weir equations
- **hs** - tailwater depth above the weir crest
- **h** - head upstream of the weir
- **n** - exponent, 1.5 for rectangular, 2.5 for a triangular weir.

For broad-crested weirs, figure 8.23 can be used to adjust the computed discharge due to submergence.

Figure 8.23 - Submergence Factors

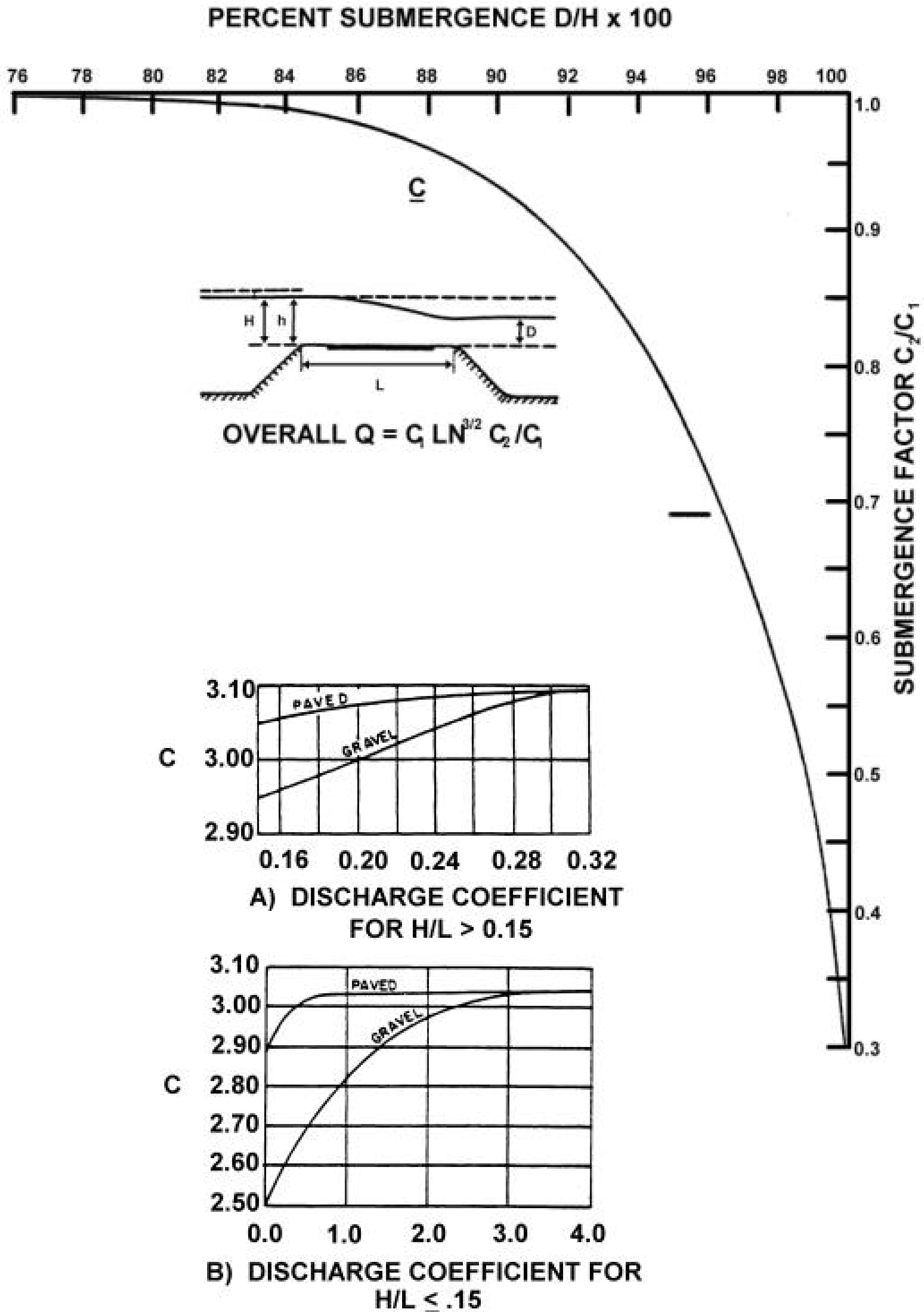


Figure 8.24 - Weir Discharge Coefficients for Road Embankments

(Source: FHWA, reference 12)

Sloping weirs

In many instances, an embankment or an emergency spillway will not be flat. In such cases it will be necessary to determine an h, to be used in the weir formula. One method that gives reasonable results involves breaking the weir into subsections and computing an average height for each of the subsections. In figure 8.25, the weir is broken into three subsections. The flow through each of the subsections is determined using:

$Q = CLH^{3/2}$

where: **H** - the average height within the subsection $(H_1 + H_2)/2$
L - length of the subsection
C - discharge coefficient

Once the flow is determined for each of the subsections, the total discharge is obtained by summing all of the subsections.

$Q_t = Q_1 + Q_2 + Q_3$

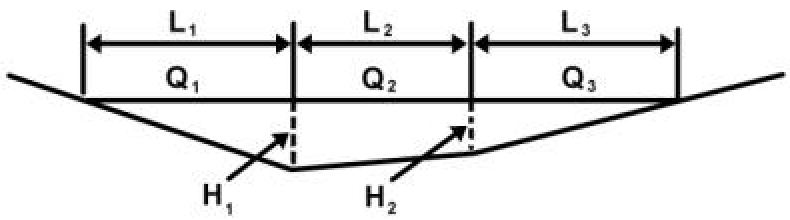


Figure 8.25 - Irregular Weir

Rating Curves

When a range of discharges are being analyzed, it will be necessary to develop a rating curve for the structure. A rating curve is a plot on a graph showing the upstream stage for a given discharge. The rating curve may be complicated if both weir flow and culvert flow will be occurring. The following steps can be used to develop a rating curve.

1. Assume an upstream water surface elevation.
2. Compute the discharge for the assumed WSEL.
3. (Compute both culvert and weir flows if each will be occurring at the assumed WSEL.)
4. Plot the results (discharge computed for the assumed WSEL).
5. Repeat until the curve is adequately defined.
6. Develop a "combined rating curve."

The rating curves for the culvert(s) and weir(s) are added together to form one rating curve. The "addition" can be done either graphically (see figure 8.28) or mathematically.

The following example will incorporate the orifice and weir equations that have been discussed, and a rating curve will be developed.

Example 8.7: Given the culvert and spillway configuration below, develop a rating curve for the structure up to 120 cfs. A tailwater rating curve computed in normal-depth example can be used to determine the tailwater.

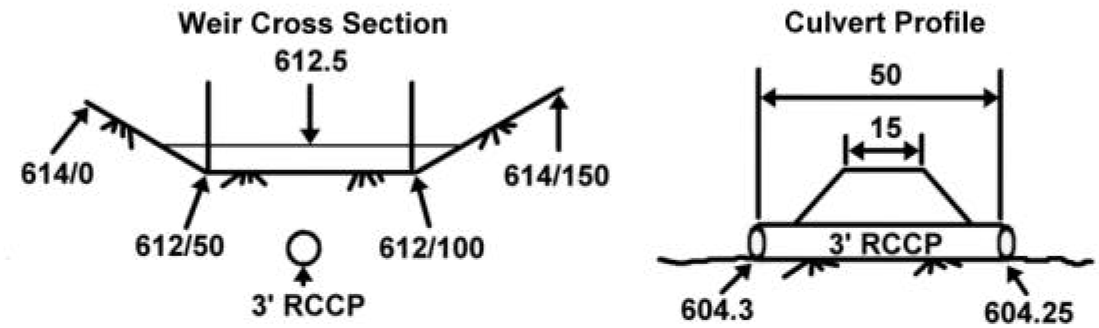


Figure 8.26 - Example Culvert and Spillway Configuration

1. Tailwater rating curve - (from example 8.1):

elevation	604.25	606.00	606.30	607.00	607.30	608.00	609.00
computed Q, cfs	0	34	50	95	120	200	425

2. Culvert Data & Rating Curve

total loss coefficient (K) = 1.81 (from example 8.3, page 124)
Area = 7.07 sq. ft. (from example 8.3)

Outlet Control computations:

equation (31): $H = K\,(Q/A)^2/2g$

$$= 1.81\,(Q/7.07)^2\,/2(32.2)$$

- rating curve based on outlet control computations

Q, cfs	34	50	80	95	120
H, ft	0.65	1.41	3.60	5.07	8.10

- inlet control (HW/D values) from nomograph (figure 8.14)

Q (cfs)	inlet control H (ft)	HW (ft)*	outlet control TWEL (ft)	H (ft)	HW (ft)*
0	0	604.3	604.3	0	604.3
34	3.0	607.3	606.0	0.65	606.7
50	4.1	608.4	606.3	1.41	607.7
80	7.7	612.0	606.8	3.60	609.4
95	9.5	613.8	607.0	5.07	612.1
120	14.4	618.7	607.3	8.10	615.4

*note: For inlet control, HW = H + 604.3 (upstream invert).
For outlet control, HW = H + TWEL.

In this example, inlet control will govern, as the computed headwater for inlet control conditions exceeds the outlet control headwater elevations. Figure 8.27 is a plot of the rating curve for the culvert.

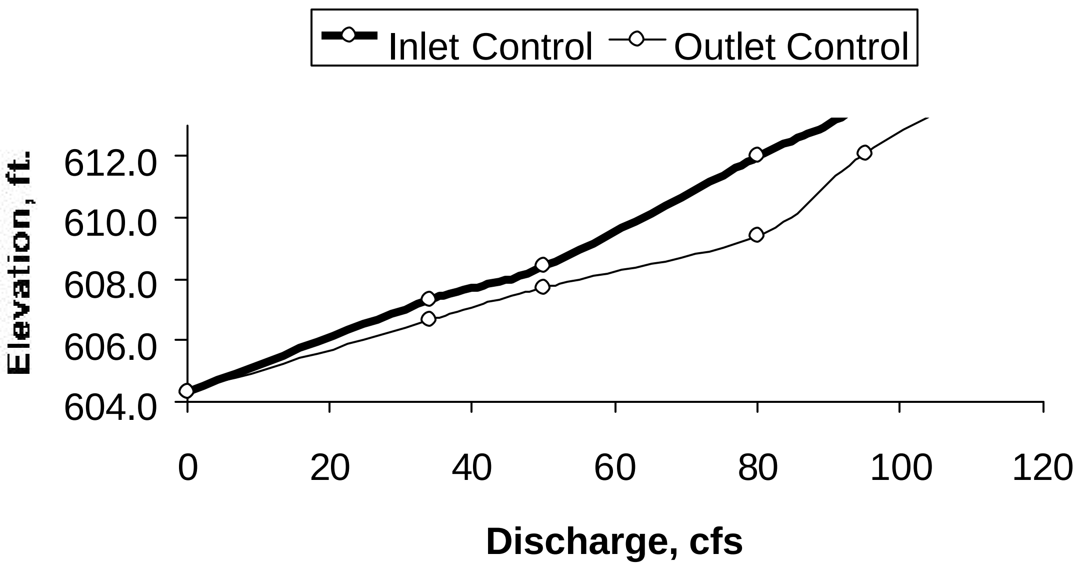


Figure 8.27 - Culvert Rating Curve

3. Weir data & rating curve

Compute discharges for various selected upstream stages using equation (36):

$Q = CLH^{3/2}$

- Select stage = **612.5 feet**, compute discharge for each subsection, (see figure 8.25).
  Select "C" values from figures 8.23 and 8.24.

$Q_{total} = Q_1 + Q_2 + Q_3 = 12.5(2.6)(.5/2)^{1.5} + 50(2.7)(.5)^{1.5} + 12.5(2.6)(.5/2)^{1.5}$

Q = **52 cfs** @ 612.5 feet

- Select stage = **613.0 feet**, compute discharge:

$Q = 25(2.7)(1/2)^{1.5} + 50(2.82)(1)^{1.5} + 25(2.7)(1/2)^{1.5} =$ **189 cfs**

Additional stages can be selected to produce the following stage-discharge relationship:

Elevation, ft	612.0	612.5	613.0	613.5	614.0
Discharge, cfs	0	52	189	403	702

4. Combined Rating Curve

Once rating curves have been developed for the culvert and the weir, it is possible to add the two curves together to obtain a combined rating curve.

Elevation, ft	604.3	607.0	609.0	610.0	611.0	612.0	612.4	612.6
Culvert, cfs	0	30	55	63	72	80	84	85
Weir, cfs	0	0	0	0	0	0	36	80
Total Q, cfs	0	30	55	63	72	80	120	165

From the rating curve, the stage upstream of the structure for a discharge of 120 cfs, is 612.4. The culvert would be carrying 84 cfs, and the weir would be carrying 36 cfs. Figure 8.28 is a plot of the combined rating curve.

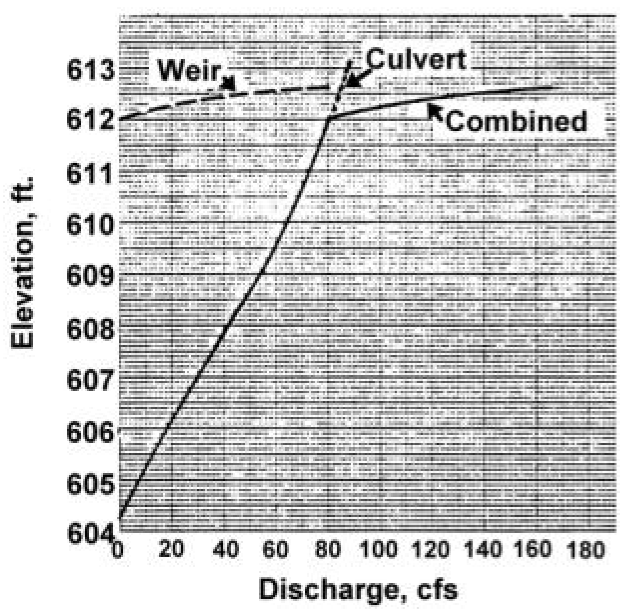


Figure 8.28 - Combined Rating Curve

Riser Pipes

In some instances, riser pipes may be used to control the outflow from a basin (see figure 8.29). The inlet will function as a weir until the weir is submerged from the effects of the downstream outlet structure (culvert). Once the weir is submerged, the culvert hydraulic will determine the rate of outflow discharge.

Following are some guidelines for the design of the riser pipe:

1. The area of the riser = 1.5 x area of the outlet pipe.
2. An anti-vortex wall is essential for the riser outlet to function as designed. For box risers, the anti-vortex wall consists of a back training wall and a dike extending from the wall to the embankment. For circular risers, the anti-vortex wall may be a splitter wall or a tangent wall (see figure 8.30). The height of these walls should be equal to the maximum headwater or two outlet pipe diameters, whichever is less.
3. The crest of the riser pipe should be a minimum distance of 2 times the diameter of the outlet pipe above the outlet invert (see figure 8.29).

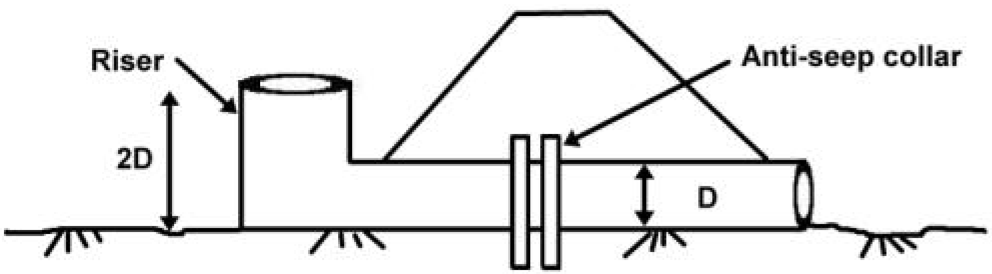


Figure 8.29 - Riser Pipe

Rectangular Riser **Circular Riser** **Circular Riser**

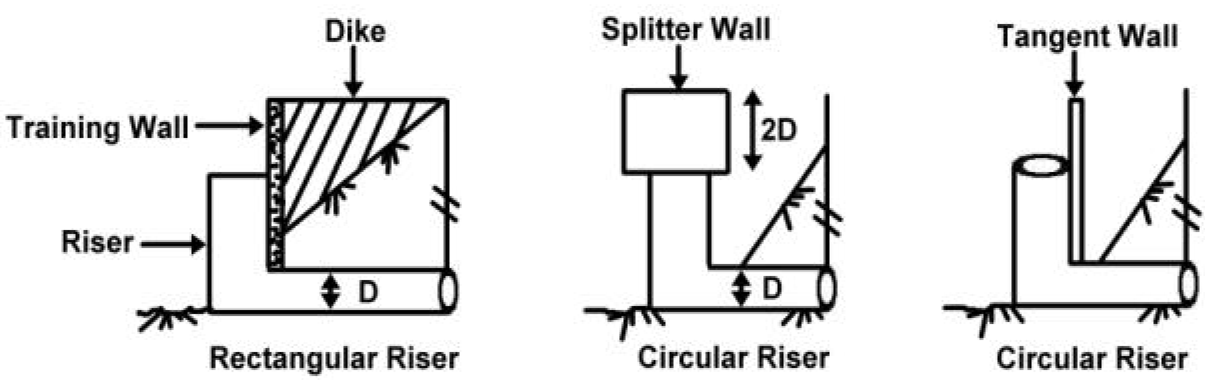


Figure 8.30 - Anti-Vortex Wall on Riser Pipe

The flow over the inlet can be determined using the weir equation:

$$Q = CLH^{3/2}$$

where: **C** - discharge coefficient, is a function of head. A value of 3.3 is reasonable for most heads on the weir. For more information on discharge coefficients for riser pipes, see reference 32.
L - weir length (see figure 8.31).
H - stage in feet above crest

The length to be used to compute the weir flow is computed to be :

L = Pi x D - obstructions (circular riser) (40)

where: **Pi** - 3.14156
D - inside diameter of pipe (cmp riser)
- inside diameter of female pipe joint (Reinforced concrete riser).

obstructions - include the thickness of the splitter wall (t).

L = W + 2B - obstructions (box riser) (41)

W - see figure 8.31
B - see figure 8.31

These are inside dimensions. If the riser lip is rounded, the dimensions are measured from the high point on the rounded lip.

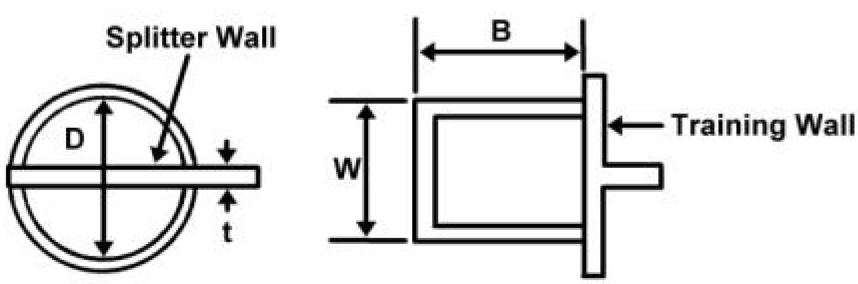


Figure 8.31 - Weir Length for Riser Pipe

As the discharge increases, the stage upstream of the riser pipe may eventually become controlled by the hydraulics of the outlet culvert. Thus, to establish a rating curve for an outlet structure that includes a riser pipe, it will be necessary to:

1. Establish a tailwater rating curve.
2. Establish a rating curve for the outlet culvert including the effects of the tailwater.
3. Establish a rating curve for the riser pipe including the effects of the backwater from the outlet culvert.
4. For a given discharge, the resulting detention-pond elevation can be obtained from the rating curve.

Example 8.8: Using the rating curve developed in example 8.7 above, develop a rating curve for a 4-foot diameter riser pipe (see figure 8.32) with crest at elevation 611.0 feet. The splitter wall is 4 inches thick.

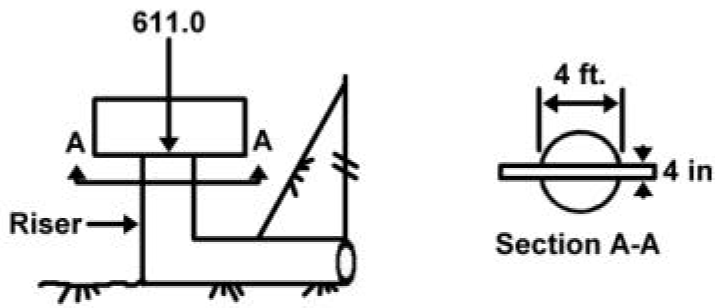


Figure 8.32 - Example Riser Pipe Problem

– **Rating Curve for Culvert** (From example 8.7)

Elevation, ft	604.3	607.0	609.0	610.0	611.0	612.0	612.4	612.6
Culvert, cfs	0	30	55	63	72	80	84	85

- Develop a Rating Curve for Riser

$Q = CLH^{3/2}$

$L = Pi \times D$ - obstructions

$= 3.3\{(Pi \times 4) - (4/12 \times 2)\}H^{3/2}$

$= 3.3\{11.9\}(Stage - 611.0)^{3/2}$

- **Rating Curve for Riser Pipe**

Stage, feet	611.0	611.5	612.0	612.4	612.6	613.0
H, feet	0.0	0.5	1.0	1.4	1.6	2.0
Q, cfs	0.0	14.0	39.0	65.0	79.0	111.0

Note: This rating curve does not include the effects of the backwater from the culvert.

- The riser pipe rating curve is plotted along with the culvert rating curve (figure 8.33).

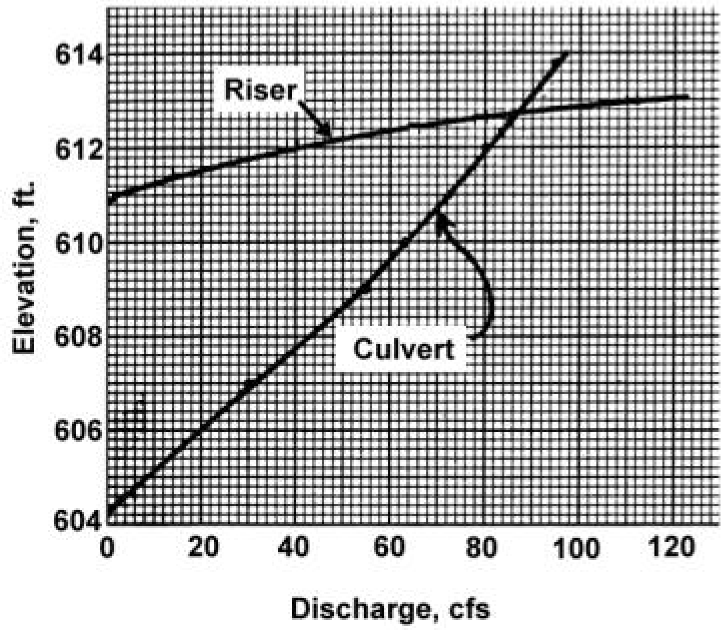


Figure 8.33 - Riser Pipe Rating Curve

From the above figure, it can be seen that at about elevation 612.8 feet, the stage on the detention pond shifts from being controlled by the riser pipe to the outlet culvert. If the backwater caused by the outlet culvert is ignored, the outlet capacity of the structure may be significantly overestimated, and the potential for structural failure will increase.